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Indian Standard

IS 13365 (Part 2) : 2019

शैल संहति का मात्रात्मक वर्गीकरण तंत्र — दिशानिर्देश सिद्धांत

भाग 2 भूमिगत द्वारो में आधार-दाब, आधार प्रणाली एवंम इंजीनियरिंग गुण के निर्धारण के लिए शैल संहति गुणता

(पहला पुनरीक्षण)

Quantitative Classification Systems of Rock Mass — Guidelines

Part 2 Rock Mass Quality for Prediction of Support Pressure, Support System and Engineering Properties in Underground Openings

(First Revision)

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Rock Mechanics Sectional Committee, CED 48

FOREWORD

The Indian Standard (Part 2) (First Revision) was adopted by the Bureau of Indian Standards, after the draft finalized by Rock Mechanics Sectional Committee had been approved by the Civil Engineering Division Council.

This standard (Part 2) covers classification of rock mass for prediction of support pressure, support system and engineering properties in underground openings. In the formulation of this standard due weightage has been given to the work of Nick Barton and his co-workers in Norwegian Geotechnical Institute (NGI) who developed this classification after studying number of case histories, as well as to the experience gathered abroad and within the country.

Quantitative classifications of rock mass have many advantages. Past experience with field tests and monitoring has provided fairly good correlations with quantitative classifications and these may be used to predict engineering behaviour of rock masses with reasonable accuracy. This is the reason why quantitative classification systems of rock mass have become very popular all over the world.

This standard has been published in four parts. The other parts in the series are:

Part 1	Rock mass rating (RMR) for predicting engineering properties
Part 3	Determination of slope mass rating
Part 4	Geological strength index (GSI)

This standard (Part 2) was first published in 1992. In this revision the title of Part 2 has also been modified from 'Rock mass quality for prediction of support pressure in underground openings' to 'rock mass quality for prediction of support pressure, support system and engineering properties in underground openings'. The revised standard gives the latest knowledge of Q system and its applications pertaining to:

a) Rock mass quality in seismic conditions;
b) Prediction of ground conditions;
c) Chart for the design of support system for tunnels;
d) Estimation of tunnel deformation of closure;
e) Critical strain; and
f) Engineering properties of rock mass.

For the purpose of eliminating the bias of an individual user, the rating for different parameters should be given in a range in preference to a single value.

For the purpose of deciding whether a particular requirement of this standard is complied with the final value, observed or calculated, expressing the result of a test or analysis shall be rounded off in accordance with IS 2 : 1960 'Rules for rounding off numerical values (*revised*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

Indian Standard

QUANTITATIVE CLASSIFICATION SYSTEMS OF ROCK MASS — GUIDELINES

PART 2 ROCK MASS QUALITY FOR PREDICTION OF SUPPORT PRESSURE, SUPPORT SYSTEM AND ENGINEERING PROPERTIES IN UNDERGROUND OPENINGS

(First Revision)

1 SCOPE

This standard (Part 2) covers the procedures for obtaining the value of rock mass quality (Q), support pressures, support system and engineering properties for both small and large underground openings.

2 REFERENCES

The standards given below contain provisions which, through reference in this text, constitute provisions of this standard. At the time of publication, the editions indicated were valid. All standards are subject to revision and parties to agreement based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards indicated below:

IS No.	*Title*
11315 (Part 11) : 1985	Method of quantitative description of discontinuities in rock mass: Part 11 Core recovery and rock quality
15026 : 2002	Tunneling methods in rock masses — Guidelines

3 PROCEDURE

3.1 Determination of the Rock Mass Quality, *Q*

The rock mass quality, Q shall be defined by the following equation:

$$Q = (RQD/J_n).\ (J_r/J_a).\ (J_w/SRF) \qquad \ldots(1)$$

where

RQD = Deere's rock quality designation $\geq$ 10 [*see* IS 11315 (Part 11)],

J_n = joint set number,

J_r = joint roughness number for critically oriented joint set,

J_a = joint alteration number for critically oriented joint set,

J_w = joint water reduction factor, and

SRF = stress reduction factor to consider *in-situ* stresses and according to the observed tunnelling conditions.

NOTE — The rock mass quality in equation 1 is related with the ultimate support pressure requirements, support system and engineering properties.

3.1.1 *Rock Quality Designation (RQD)*

RQD shall be defined as the sum of core pieces with lengths greater or equal to 10 cm expressed as percentage of length of the borehole. In the absence of a borehole, RQD may be estimated from the number of joints per unit volume of the rock mass, in which the number of joints per meter for each joint set are added. Thus, RQD for clay free rock masses shall approximately be given by the following formula:

$$RQD = 115 - 3.3\ J_v \leq 100 \quad [\ (see\ \text{IS11315 (Part 11)}\ ] \qquad \ldots(2)$$

$$RQD = 110 - 2.5\ J_v \leq 100 \quad (\text{for } J_v \text{ between 4 and 44}) \qquad \ldots(3)$$

where

J_v = total number of joints per m^3 called volumetric joint count.

Equation 2 is for the elongated shape or flat rock blocks, while equation 3 is for rock blocks of cubical shape.

If RQD is less than 10 percent a minimum value of 10 is used to evaluate rock mass quality (Q) (*see* Table 1).

Table 1 Rock Quality Designation RQD

(*Clause* 3.1.1)

Sl No.	Classification	RQD
(1)	(2)	(3)
i)	A-Very poor	0 – 25
ii)	B-Poor	25 – 50
iii)	C-Fair	50 – 75
iv)	D-Good	75 – 90
v)	E-Excellent	90 – 100

NOTES

1 Where RQD is reported or measured as $\leq$ 10 (including 0), a minimum value of 10 is used to evaluate Q in equation 1.

2 RQD intervals of 5, that is, 100, 95, 90, etc, are sufficiently accurate.

3.1.2 *Joint Set Number (J_n)*

The parameter (J_n) representing the number of joint sets shall often be affected by foliations, schistocity, slaty cleavages or beddings, etc. If strongly developed, these parallel discontinuities should be counted as a complete joint set. If there are few joints visible or only occasional breaks in rock core due to these features, then it should be counted as 'random joints' while evaluating J_n from Table 2.

Table 2 Joint Set Number, J_n

(*Clause* 3.1.2)

Sl No.	Classification	J_n
(1)	(2)	(3)
i)	A - Massive, no or few joints	0.5 – 1.0
ii)	B - One joint set	2
iii)	C - One joint set plus random	3
iv)	D - Two joint sets	4
v)	E - Two joint sets plus random	6
vi)	F - Three joint sets	9
vii)	G - Three joint sets plus random	12
viii)	H - Four or more joint sets, random, heavily jointed, sugar cube, etc	15
ix)	I - Crushed rock, earth-like	20

NOTES

1 For tunnel intersections, use 3 × J_n.

2 For portals, use 2 × J_n.

3.1.3 *Joint Roughness Number and Joint Alteration Number (J_r and J_a)*

The parameters J_r and J_a, given in Tables 3 and 4 represent roughness and degree of alteration of joint walls or filling materials, respectively. The parameters J_r and J_a should be relevant to the weakest and critically oriented joint set or clay filled discontinuity in a given zone [For critically oriented joint set, *see* IS 13365 (Part 1)]. If the joint set or discontinuity with the minimum value of (J_r/J_a) is favourably oriented for stability, then a second less favourably oriented joint set or discontinuity may be of greater significance, and its higher value of (J_r/J_a) should be used when evaluating *Q* from Equation 1.

3.1.4 *Joint Water Reduction Factor (J_w)*

The parameter J_w (*see* Table 5) is a measure of water pressure, which has an adverse effect on the shear strength of joints. This is due to reduction in the effective normal stress across joints. Water may in addition cause softening and possible outwash in the case of clay filled joints. The value of J_w should correspond to the anticipated future ground water (hydro-geological) condition where seepage erosion or leaching of chemical can alter permeability of the rock mass significantly.

3.1.5 *Stress Reduction Factor (SRF)*

The parameter SRF (*see* Table 6) is a measure of the following:

a) Loosening pressure in the case of an excavation through shear zones and clay bearing rock masses,

b) Rock stress $\frac{q_c}{\sigma_1}$ in a competent rock mass where is uniaxial compressive strength of intact rock material and is the major principal stress before excavation, and

c) Squeezing or swelling pressures in incompetent rock masses. *SRF* can also be regarded as a total stress parameter.

NOTE — SRF ratings for squeezing rock conditions are not found reliable for predicting support pressures as these depend upon tunnel wall displacements.

3.2 As seen from equation 1, the rock mass quality (*Q*) may be considered a function of only three parameters which are crude measures of the following:

a) Block size (*RQD*/ J_n) : It represents overall structure of rock mass

b) Inter block shear strength (J_r/J_a) : It has been found that $\tan^{-1}[(J_w.J_r/J_a) + 0.1]$ is a fair approximation to the actual peak sliding angle of friction along the clay coated joints

c) Active stress (J_w /S*R*F) : It is an empirical factor describing the effective active stress

Table 3 Joint Roughness Number, J_r

(*Clause* 3.1.3)

Sl No.	Classification	J_r
(1)	(2)	(3)
i)	***Rock-wall contact, and***	
ii)	***Rock-wall contact before 10 cm of shear movement***	
	A - Discontinuous joint	4.0
	B - Rough or irregular, undulating	3.0
	C - Smooth, undulating	2.0
	D - Slickensided, undulating	1.5
	E - Rough, irregular, planar	1.5
	F - Smooth, planar	1.0
	G - Slickensided, planar	0.5
iii)	***No rock-wall contact when sheared***	
	H - Zone containing clay minerals thick enough to prevent rock-wall contact	1.0
	J - Sandy, gravelly or crushed zone thick enough to prevent rock wall contact	1.0

NOTES

1 Descriptions refer to small-scale features and intermediate scale features, in that order.

2 Add 1.0 if the mean spacing of the relevant joint set is greater than 3m.

3 J_r = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are oriented in the estimated sliding direction.

3.3 Collection of Field Data

The length of core or an exposed excavation to be used for evaluating the first four parameters (*RQD*, J_n, J_r and J_n) shall depend on the uniformity of the rock mass. If there is little variation, a core or wall length of 5 to 10 m should be sufficient. However, in a few meters wide closely jointed shear zone with alternate sound rock, it shall be necessary to evaluate these parameters separately, if it is considered likely that the closely jointed shear zones are wide enough to justify special treatment (that is, additional shotcrete) compared to only systematic bolting in the remainder of the excavation. If on the other hand the shear zones are narrow than 1 to 2 m and occur frequently, then an overall reduced value of *Q* may be most appropriate since increased support is likely to be applied uniformly along the entire length of such variable zones. In such cases a core or wall length of 10 to 50 m may be needed to obtain an overall picture of the reduced rock mass quality. This classification may not be applicable to thinly layered rock mass or very soft rock mass with q_c< 1 MPa which may be classified as soil.

NOTES

1 Values of the rock mass quality *Q* should be obtained separately for the roof, floor and the two walls, particularly when the geological description of the rock mass is not uniform around the periphery of an underground opening.

2 Mean value of rock mass quality should be taken as root mean square value of its maximum and minimum values.

3 In case of power tunnels, the value of J_w for calculation of ultimate support pressures should be reduced assuming that seepage water pressure in Table 5 is equal to the internal water pressure in the future.

Table 4 Joint Alteration Number, J_a

(*Clause* 3.1.3)

Sl No.		Classification	J_a	ϕ_r (Degree) (Approx)
(1)		(2)	(3)	(4)
i)	**Rock wall contact (No mineral filling, only coating)**			
	A	Tightly healed, hard, non-softening, impermeable filling, that is quartz or epidote	0.75	–
	B	Unaltered joint walls, surface staining only	1.0	25 - 35°
	C	Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc	2.0	25 - 30°
	D	Silty or sandy clay coatings, small clay fraction (non-softening)	3.0	20 - 25°
	E	Softening or low friction clay mineral coatings, that is kaolinite, mica. Also chlorite, talc, gypsum, and graphite, etc and small quantities of swelling clays (Discontinuous coatings, 1-2 mm or less in thickness)	4.0	8 - 16°
ii)	**Rock wall contact before 10 cm shear (Thin mineral fillings)**			
	F	Sandy particles, clay-free disintegrated rock, etc.	4.0	25 - 30°
	G	Strongly over-consolidated, non-softening clay mineral fillings (continuous, < 5 mm in thickness)	6.0	16 - 24°
	H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, < 5 mm in thickness)	8.0	12 - 16°
	J	Swelling clay fillings, that is montmorillonite (continuous, < 5 mm in thickness). Value of J_a depends on per cent of swelling clay-size particles, and access to water, etc	8 - 12	6 - 12°
iii)	**No rock wall contact when sheared (Thick mineral fillings)**			
	K	Zones or bands of disintegrated or crushed rock, strongly over-consolidated	6.0	16 - 24°
	L	Zones or bands of clay, disintegrated or crushed rock, Medium or low over-consolidation or softening fillings	8.0	12 - 16°
	M	Zones or bands of clay, disintegrated or crushed rock. Swelling clay J_a depends on percent of swelling clay-size particles	8 - 12	6 - 12°
	N	Thick, continuous zones or bands of clay, strongly over- consolidated	10	12 - 16°
	O	Thick, continuous zones or bands of clay, Medium or low over-consolidation	13	12 - 16°
	P	Thick, continuous zones or bands with clay. Swelling clay. J_a depends on percent of swelling clay-size particles	13 - 20	6 - 12°

NOTE — Values of ϕ_r are intended as an approximate guide to the mineralogical properties of the alteration products, if present.

Table 5 Joint Water Reduction Factor, J_w

(*Clause* 3.1.4)

Sl No.		Classification	Approx. Water Pressure MPa	J_w
(1)		(2)	(3)	(4)
i)	A	Dry excavations or minor inflow, that is 5 lt./min locally	< 0.1	1
ii)	B	Medium inflow or pressure occasional out-wash of joint fillings	0.1 - 0.25	0.66
iii)	C	Large inflow or high pressure in competent rock with unfilled joints	0.25 - 1.0	0.5
iv)	D	Large inflow or high pressure, considerable out-wash of joint fillings	0.25 - 1.0	0.33
v)	E	Exceptionally high inflow or water pressure at blasting, decaying with time	> 1.0	0.2 - 0.1
vi)	F	Exceptionally high inflow or water pressure continuing without noticeable decay	> 1.0	0.1 - 0.05

NOTES

1 Factors C to F are crude estimates. Modify J_w, if the rock mass is drained or grouting is carried out.

2 Special problems caused by ice formation are not considered.

3.4 Classification of the Rock Mass

On the basis of the *Q*-value, the rock masses may be classified into the following nine categories (*see* Table 7).

3.5 Rock Mass Quality in Seismic Conditions

For the designs of tunnels and other underground openings in seismic conditions, rock mass quality for seismic conditions ($Q_{seismic}$) has been suggested as follows:

$$Q_{seismic} = D^{-0.3}\, Q/\,(1+\alpha_v)^3 \quad \dots (4)$$

where

α_v = coefficient of peak vertical ground acceleration (PGA) at tunnel depth,

= 0.185 or measured value;

D = tunnel span or height, whichever is more;

< $\lambda/4$ to avoid resonance;

λ = wavelength of seismic waves; and

Q = actual post-construction rock mass quality.

Strong impact of seismic waves is expected on tunnel supports when the ratio of seismic wavelength (λ) to tunnel span (D) is between 1.0 and 4.0, particularly near thick shear zones and fault zones.

3.6 Estimating Support Pressures

All methods of tunnel excavation and support systems presently used, allow some degree of deformation of the surrounding rock mass. In most of the poorer qualities of rock masses (excluding squeezing and swelling conditions), the final support pressures may tend to be somewhat greater if the temporary support is excessively flexible or if the installation of support is delayed. Use of immediate shotcrete and/or rock bolt as temporary supports tends to minimize the final pressures as compared to steel rib support. This is because the shotcrete and/or rock bolt allows just sufficient deformation for arching to develop but prevents loosening of the rock mass within this arch.

It may be mentioned that *Q* referred to in the following correlation is actually the post excavation quality of a rock mass. In tunnels, the geology of the rock mass is usually studied after blasting and on the spot decision is taken on support density and spacing of steel ribs, etc.

3.6.1 *Non-squeezing Ground ($H < 350\, Q^{0.33}$)*

3.6.1.1 *Ultimate roof support pressure*

a) *Vertical support pressure* — The ultimate roof support pressure is related to ultimate roof rock mass quality (Q_u) by the following empirical correlation for tunnels and caverns with arched roof:

$$p_{ru} = \frac{0.2}{J_r} . Q_u^{-0.33} . \mathrm{f} \quad \text{MPa} \qquad \dots (5)$$

where

p_{ru} = ultimate roof support pressure, in MPa;

J_r = joint roughness number;

Q_u = ultimate (post-construction) rock mass quality = Q in roof;

f = correction factor for overburden or tunnel depth

= 1 + (H-320)/800; … (6)

≥ 1 ; and

H = overburden above crown of tunnel or tunnel depth below ground level; in metres.

NOTE — Q_u may reduce with time due to weathering and seepage particularly in rocks showing no reaction during 1st wetting-drying-cycle and cracking and/or beginning of decay up to 50 percent of the original mass up to 3rd cycle in slake durability test and the rocks having UCS of intact rock material (q_c <50 MPa).

b) *Ultimate wall support pressure* – In view of the more favourable position of walls as compared to roofs, the following increased value of wall rock quality (Q_{wu}) may be used to find out ultimate wall support (horizontal support) pressure on the walls:

1) In good to exceptionally good qualities of rock masses of group 1 (Q_u > 10)

$$Q_{wu} = 5Q_u \qquad \dots (7)$$

2) In very poor to fair qualities of rock masses of group 2 ($0.1<Q_u<10$)

$$Q_{wu} = 2.5Q_u \qquad \dots (8)$$

Table 6 Stress Reduction Factor *SRF*

(*Clause* 3.1.5)

Sl No.		Classification			*SRF*
i)	Weakness zones intersecting the underground opening, which may cause loosening of rock mass				
	A	Multiple occurrence of weak zones within a short section containing clay or chemically disintegrated rock, very loose surrounding rock (any depth), or long sections with incompetent (weak) rock (any depth). For Squeezing see M and N			10
	B	Multiple shear zones within a short section in competent clay-free rock with loose surrounding rock (any depth)			7.5
	C	Single weak zone with or without clay or chemically disintegrated rock (depth ≤ 50 m)			5
	D	Loose, open joints, heavily jointed or 'sugar cube' etc (any depth)			5
	E	Single weak zone with or without clay or chemically disintegrated rock (depth > 50 m)			2.5
	NOTE — Reduce these values of *SRF* by 25-50 percent if the weak zones only influence but do not intersect the underground opening.				
ii)	Competent, mainly massive rock, stress problem		q_c/σ_1	σ_θ/q_c	*SRF*
	F	Low stress, near surface, open joints	> 200	< 0.01	2.5
	G	Medium stress, favourable stress condition	200 - 10	0 - 01 - 0.3	1
	H	High stress, very tight structure. Usually favourable to stability. May also be unfavourable, stability dependent on the orientation of stresses compared to jointing/weakness planes	10 - 5	0.3 - 0.4	0.5 - 2 2 - 5
	J	Moderate slabbing and /or slabbing after > 1 h in massive rock	5 - 3	0.5 - 0.65	5 - 50
	K	Spalling or rock burst after a few minutes in massive rock	3 - 2	0.65 - 1	50 - 200
	L	Heavy rock burst and immediate dynamic deformation in massive rock	< 2	> 1	200 - 400
	NOTES **1** For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce q_c to 0.75 q_c; when $\sigma_1/\sigma_3 > 10$, reduce q_c to 0.50 q_c (where q_c = unconfined compressive strength, σ_1 and σ_3 are major and minor principal stresses, and σ_θ= maximum tangential stress (estimated from elastic theory). **2** When the depth of the crown below surface is less than span: suggest *SRF* increase from 2.5 to 5 for such cases (*see* F)				
iii)	Squeezing rock: plastic deformation in incompetent rock under the influence of high pressure			σ_θ/q_c	*SRF*
	M	Mild squeezing rock pressure		1 - 5	5 - 10
	N	Heavy squeezing rock pressure		> 5	10 - 20
	NOTE — Cases of squeezing rock may occur for tunnel depth $H > 350Q^{1/3}$ [*Q* with *SRF* = 2.5] in meters. Also see Section 3.8 for prediction of ground condition using rock mass number N.				
iv)	Swelling rock: Chemical swelling activity depending on the presence of water				*SRF*
	O	Mild swelling rock pressure			5 - 10
	P	Heavy swelling rock pressure			10 - 20

Table 7 Classification of Rock Mass Based on *Q* - Values

(*Clause* 3.4)

Sl No. (1)	*Q* (2)	Group (3)	Classification (4)
i)	10 – 40	1	Good
	40 – 100		Very good
	100 – 400		Extremely good
	400 – 1 000		Exceptionally good
ii)	0.1 – 1	2	Very poor
	1 – 4		Poor
	4 – 10		Fair
iii)	0.001 – 0.01	3	Exceptionally poor
	0.01 – 0.1		Extremely poor

3) In extremely poor and exceptionally poor qualities of rock masses of group 3 ($Q_u < 0.10$)

$$Q_{wu} = Q_u \qquad \ldots(9)$$

The ultimate wall support pressure (p_{wu}) is given by the following correlation, in MPa:

$$p_{wu} = \frac{0.2}{J_r} . Q_{wu}^{-0.33} . \mathrm{f} \quad \text{MPa} \qquad \ldots (10)$$

where

Q_{wu} = ultimate wall rock quality.

3.6.1.2 *Short-term support pressure*

Temporary supports should be designed for short-term support pressure only. These supports are strengthened later on by shotcreting or lining to take care of ultimate support pressures.

a) *Short-term roof support pressure* — The short-term roof support pressure is related with the so called short-term roof rock quality (Q_{ri}) by the following correlation for tunnels and caverns with arched roof:

$$p_{ri} = \frac{0.2}{J_r} . Q_{ri}^{-0.33} . \mathrm{f} \quad \text{MPa} \qquad \ldots (11)$$

where

p_{ri} = short-term (vertical) roof support pressure, in MPa; and

Q_{ri} = $5Q$ = short-term (post-construction) rock mass quality;

Field observations of the support pressures are close to those estimated from Equation 11 for non-squeezing rock conditions.

b) *Short-term wall support pressure* — The short-term wall support pressure (p_{wi}) is similarly given by the following equation, in MP$_a$:

$$p_{wi} = \frac{0.2}{J_r} . Q_{wi}^{-0.33} . \mathrm{f} \quad \text{MPa} \qquad \ldots (12)$$

where

Q_{wi} = short-term (post-construction) wall rock mass quality.

The short-term wall rock quality Q_{wi} is obtained after multiplying Q by a factor which depends on the magnitude of Q as given below:

a) $Q > 10$: $Q_{wi} = 5Q_{ri} = 25Q$

b) $0.1 < Q < 10$: $Q_{wi} = 2.5Q_{ri} = 12.5Q$

c) $Q < 0.10$: $Q_{wi} = 1.0Q_{ri} = 5Q$

For rock mass quality (Q) and $Q_{seismic}$ used in equations 1 to 12, the corresponding support pressure may also be obtained from the chart given in Fig. 1. The shaded envelope is the estimate of range of support pressures to be expected in practice.

NOTES

1 The ratio of ultimate to short-term support pressure may increase from 1.7 as assumed in **3.6.1.1** and **3.6.1.2** to as high as 6 times in cases of rock mass with soluble and erodible joint fillings and seepage problems. Concrete lining should be designed for high net ultimate support pressures and future ground water pressure in the roof in such rock masses.

2 The observed short-term wall support pressure may not be significant in non-squeezing rock conditions. It is, therefore, recommended that these may be neglected in the case of tunnels in rock masses of good quality of group 1 ($Q > 10$).

3 Although the bottom support pressure is negligible in non-squeezing ground conditions, invert support may be used when the estimated wall support pressure requires the use of wall support in exceptionally poor rock conditions and squeezing ground conditions.

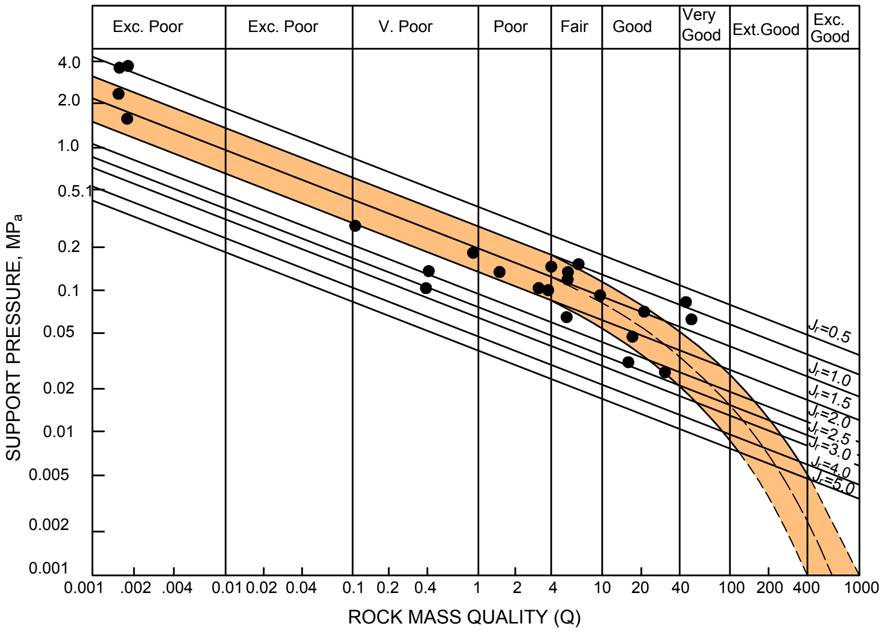


FIG. 1 RELATIONSHIP BETWEEN Q AND SUPPORT PRESSURE

3.6.2 *Squeezing Ground ($H > 350\ Q^{0.33}$ and $Jr/Ja < ½$)*

3.6.2.1 *Ultimate support pressure*

a) *Roof support pressure* — The ultimate roof support pressure (p_{ru}) is related to ultimate roof (post-construction) rock mass quality (Q_u) by the following empirical correlation:

$$p_{ru} = \frac{0.2}{J_r} . Q_u^{-0.33} . \mathrm{f.f'} \quad \text{MPa} \qquad \dots (13)$$

where

f = correction factor for overburden or tunnel depth (equation 6), and

f' = correction factor for tunnel closure (*see* Table 8 and Fig. 2).

Values of correction factors for tunnel closure (f') may be obtained from Table 8, the basis of design value of tunnel closure.

b) *Wall support pressure* — The ultimate wall support pressure (p_{wu}) is obtained by the following empirical correlation:

$$p_{wu} = \frac{0.2}{J_r} . Q_{wu}^{-0.33} . f . f' \quad \text{MPa} \qquad \dots (14)$$

where

Q_{wu} should be used as per **3.6.1.1** (b).

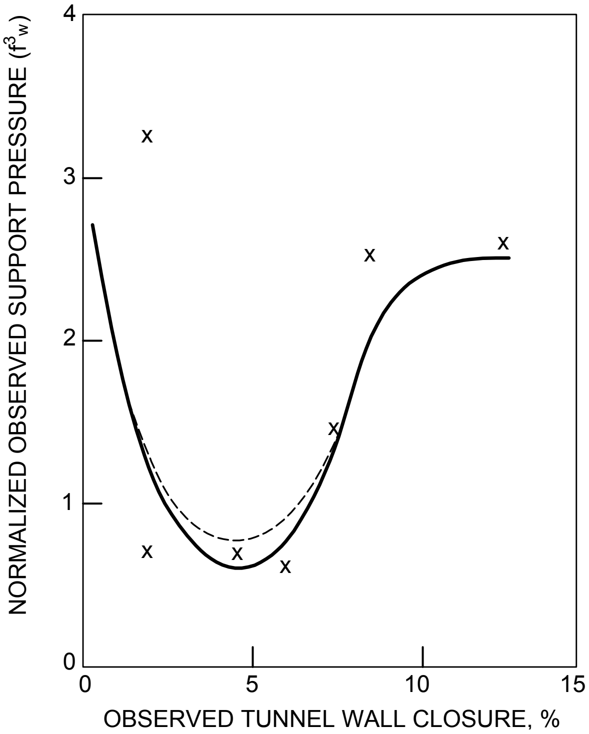


FIG. 2 CORRECTION FACTOR FOR TUNNEL CLOSURE UNDER SQUEEZING GROUND CONDITION ($H > 350\ Q^{0.33}$ m AND $J_r/J_a < 1/2$)

Table 8 Correction Factor *f'* for Tunnel Closure

(*Clause* 3.6.2)

Sl No.	Rock Condition	Support System	Tunnel Closure (u_a/a) percent	Correction Factor f'
(1)	(2)	(3)	(4)	(5)
i)	Non-squeezing ($H < 350\ Q^{0.33}$)	–	< 1 or ε_{cr} (critical strain, *see* **6**)	1.0
ii)	Squeezing ($H > 350\ Q^{0.33}$, $J_r/J_a < 0.5$)	Very stiff	< 2	> 1.8
iii)	-do-	Stiff	2 - 4	0.85
iv)	-do-	Flexible	4 - 6	0.70
v)	-do-	Very flexible	6 - 8	1.15
vi)	-do-	Extremely flexible	> 8	1.8

Notation: u_a = radial tunnel closure; a = tunnel radius

NOTES

1 Tunnel closures more than 4 percent of tunnel span should not be allowed otherwise support pressures are likely to build up rapidly due to loosening of rock mass. In such cases, additional rock anchors should be installed immediately to arrest the tunnel closure within limiting value. Otherwise the ratio of ultimate to short-term support pressure may rise to 2 to 3 times.

2 Steel ribs with struts may not absorb more than 2 percent tunnel closure.

3 Tunnels in highly squeezing ground should be less than 6 m in span to ensure better rate of tunneling with less construction problems. Tunnel instrumentation is recommended.

4 In case of very good and hard rocks, rock burst may occur in place of squeezing when overstressed ($H > 900$ m and $J_r/J_a > ½$)

5 In case of parallel tunnels, the wall support pressure shall be much higher than roof support pressure, if clear spacing between tunnels is less than sum of the span of openings.

3.6.2.2 *Short-term support pressure*

a) *Short-term roof support pressure* — The short-term roof support pressure is related with the short-term (post-construction) rock mass quality (Q_i) by the following correlation:

$$p_{ri} = \frac{0.2}{J_r} . Q_{ri}^{-0.33} . \text{f.f}' \quad \text{MPa} \qquad \dots (15)$$

Q_i = $5Q$ = immediate or short-term (post-construction) rock mass quality.

b) *Short-term wall support pressure* — The short-term wall support pressure (P_{wi}) is similarly given by the following equation:

$$p_{wi} = \frac{0.2}{J_r} . Q_{wi}^{-0.33} . \text{f.f}' \quad \text{MPa} \qquad \dots (16)$$

where

Q_{wi} = immediate or short-term (post-construction) wall rock mass quality [*see* **3.6.1.2** (b)].

3.6.2.3 *Support pressure for shafts*

Above equations using Q_w should be used for estimating the wall support pressures in circular and rectangular shafts.

3.7 Unsupported Span

The equivalent dimension (D_e) of self supporting or unsupported tunnel is given below:

$$D_e = 2\, Q^{0.4} \qquad \dots(17)$$

where

D_e = limiting value of span, diameter or height, in metre/ESR;

Q = rock mass quality (post-construction); and

ESR = excavation support ratio (*see* Table 9).

In equivalent dimension, the span or diameter is used when analyzing the roof support, and the diameter or height for the wall support.

Excavation support ratio (*ESR*) appropriate to a variety of underground excavations is listed in Table 9 in increasing order of importance or degree of safety.

For working out the temporary support requirements, *ESR* values mentioned above should be multiplied by a factor 1.5 and *Q* should be increased to 5*Q*.

General requirements for permanently unsupported openings are:

$$J_n < 9,\ J_r > 1.0,\ J_a < 1.0,\ J_w = 1.0,\ SRF < 2.5$$

Further, conditional requirements for permanently unsupported openings shall be as follows:

a) If $RQD < 40$, need $J_n < 2$;

b) If $J_n = 9$, need $J_r > 1.5$ and $RQD > 90$;

c) If $J_r = 1.0$, need $J_w < 4$;

d) If $SRF > 1$, need $J_r > 1.5$;

e) If span > 10 m, need $J_n < 9$; and

f) If span > 20 m, need $J_n < 4$ and $SRF < 1$.

The above general and conditional requirements also hold for 'self-supporting' equation shown in, Table 10 Sl No. i).

3.8 Prediction of Ground Conditions

The knowledge of expected ground condition for tunnelling plays an important role in the selection of excavation method and designing a support system. The tunnelling media could be stable/competent (and or non-squeezing) or squeezing/failing depending upon the in situ stress and the rock mass strength. A weak over-stressed rock mass would experience squeezing ground condition, whereas a hard and massive over-stressed

Table 9 Values of Excavation Support Ratio (*ESR*)

(*Clause* 3.7)

Sl No.		Type of Excavation	*ESR*
(1)		(2)	(3)
i)	A	Temporary mine openings, etc	3 - 5
ii)	B	Vertical shafts[1]:	
		a) circular sections	2.5 Approx.
		b) rectangular/square section	2.0 Approx.
iii)	C	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), water supply tunnels, pilot tunnels, drifts and headings for large openings	1.6
iv)	D	Minor road and railway tunnels, surge chambers, access tunnels, sewage tunnels, etc	1.3
v)	E	Power houses, storage rooms, water treatment plants, major road and railway tunnels, civil defence chambers, portals, intersections, etc	1.0
vi)	F	Underground nuclear power stations, railway stations, sports and public facilities, factories, etc	0.8
vii)	G	Very important caverns and underground openings with a long lifetime, ≈ 100 years, or without access for maintenance	0.5

[1] Dependent of purpose. May be lower than given values.

rock mass may experience rock burst condition. On the other hand, when the rock mass is not over-stressed, the ground condition is termed as stable or competent (non-squeezing) for the same rock mass quality, provided there is no structurally controlled instability.

Tunnelling in the competent ground conditions can again face two situations:

a) Where no supports are required, that is a self-supporting condition; and
b) Where supports are required for stability; which is called non-squeezing condition.

The squeezing ground condition has been divided into three classes on the basis of tunnel closures as minor squeezing (u_a/a = 1-3percent), severe squeezing (u_a/a = 3-5 percent) and very severe squeezing (u_a/a > 5 percent) [(Here u_a is radial tunnel closure or deformation) and 'a' is tunnel radius]. Equations given in Table 10 shall be used to predict/estimate the ground condition for tunnelling. In Table 10, rock mass number *N* is obtained by taking the *SRF* rating as 1 (constant value for all conditions) in Equation 1, that is $N = Q$ (with $SRF = 1$). '*B*' and '*H*' in Table 10 is tunnel width or span and tunnel depth respectively, in metres. Once the ground condition is known using the Equations given in Table 10, the rating for *SRF* shall be picked accordingly from Table 6 and then estimate Q in advance of tunnelling. This has helped in avoiding the uncertainty in obtaining appropriate *SRF* ratings in rock mass quality *Q* (Equation 1). Thus, *N* is complimentary to *Q*.

Table 10 Prediction of Ground Condition Using Rock Mass Number N

(*Clause* 3.8)

Sl No.	Ground Conditions	Correlations for Predicting Ground Condition
(1)	(2)	(3)
i)	Self-supporting	$H < 23.4\ N^{0.88}.\ B^{-0.1}$ and $1\,000\ B^{-0.1}$ and $B < 2\ Q^{0.4}$ m
ii)	Non-squeezing ($u_a/a < 1$ percent)	$23.4\ N^{0.88}.B^{-0.1} < H < 275\ N^{0.33}.\ B^{-0.1}$
iii)	Minor squeezing (u_a/a = 1-3 percent)	$275\ N^{0.33}.\ B^{-0.1} < H < 450\ N^{0.33}.\ B^{-0.1}$ and $J_r\ /J_a<0.5$
iv)	Severe squeezing (u_a/a = 3-5 percent)	$450\ N^{0.33}.\ B^{-0.1} < H < 630\ N^{0.33}.\ B^{-0.1}$ and $J_r\ /J_a < 0.5$
v)	Very severe squeezing (u_a/a > 5 perceents)	$H > 630\ N^{0.33}.\ B^{-0.1}$ and $J_r\ /J_a < 0.25$
vi)	Mild Rock Burst	$H.B^{0.1} > 1\,000$ m, $J_r/J_a > 0.5$, $N > 1.0$

4 CHART FOR SUPPORT SYSTEM

The *Q* value is related to tunnel support requirements with the equivalent dimensions of the excavation. The relationship between *Q* (post-construction rock mass quality) and the equivalent dimension of an excavation determines the appropriate support measures as depicted in Fig. 3. The thickness of shotcrete of steel fibre reinforced shotcrete (*SFRS*) and spacing of rock bolts should be selected according to *Q* and tunnel width in case of roof. The length of rock bolt is selected on the basis of tunnel width. In case of walls of caverns and tunnels, the thickness of shotcrete or *SFRS* should be selected according to Q_w and the height of cavern and tunnel. Similarly, the length of rock bolts should be selected on the basis of height of cavern and tunnel. The upper curve of Fig. 3 gives a permissible limit of width of cavern for a given *Q* value for ease of construction. $Q_{seismic}$ (*see* equation 4) shall be used in Fig. 3 in place of *Q* for designing the support in case of tunnels in seismically active regions. In dynamic conditions the rock bolts shall have full-column resin grout and the reinforced concrete lining shall have good bond with the rock mass to reduce the bending stress in the lining. Chemical grouting may be used for contact grouting of tunnel lining and rock mass when cement does not bond well with some rocks. Residual strains are accumulated with each minor and major earthquake around openings. Thus support pressure keeps on building up with time. Dynamic support pressures are observed to be more near shear zones and active faults in Himalaya. IS 15026 should be referred for further details.

Fore poling is not directly included in the support chart related to the *Q*-system. However, the suggested rock support in rock masses with low *Q*-values is based on the use of fore poling during excavation. The need for fore poling is dependent on the span of the underground opening and the rock mass quality. Normally, fore poling is used in combination with reduced excavation length of the blast rounds and/or excavation of only a part of the cross section of the excavation. Generally, it is suggested to use fore poling in rock masses with *Q*-values lower than 0.1 to 0.6, depending on the span of the underground opening. The spacing between the fore poles is normally around 0.3 m (0.2 to 0.6 m).

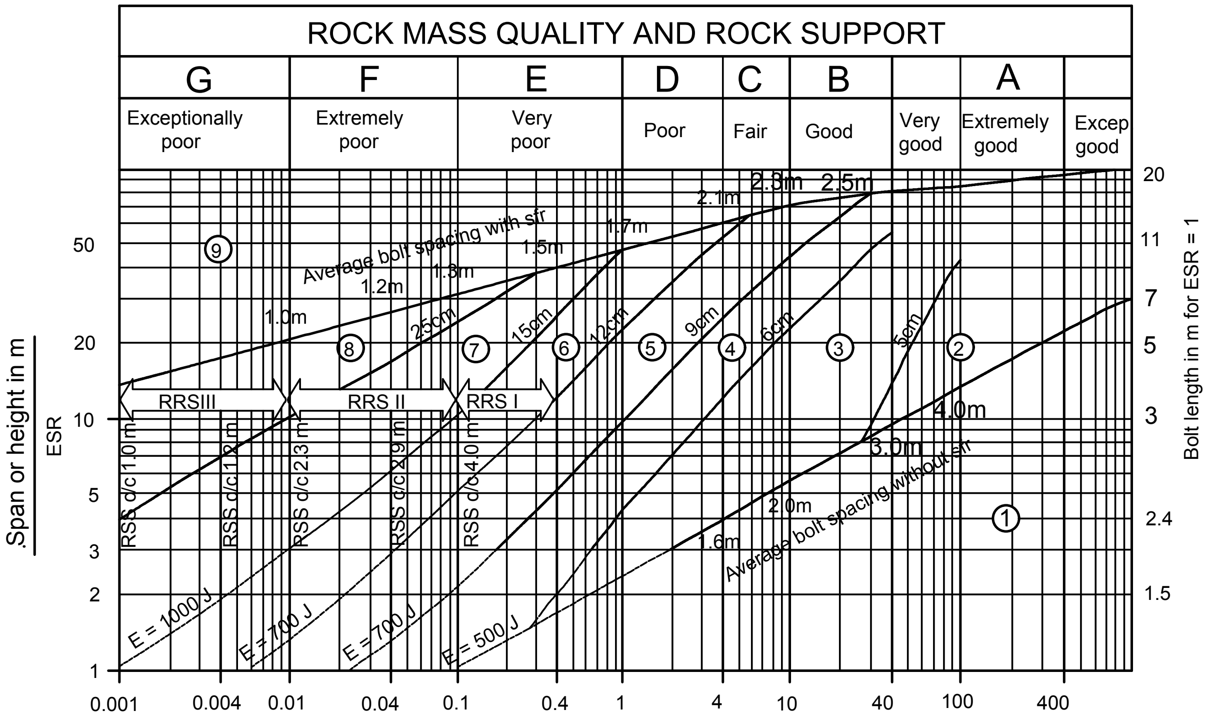
NOTES

1 Figure 3 may be useful for preliminary design of supports. Supports may be validated using numerical modeling.

2 Figure 3 may be applicable for tunnels having diameter up to 10 m. But for chambers/caverns/ large tunnels in weak rocks like in Himalaya, support system shall be finalized with numerical modeling.

3 It is experienced that Fig. 3 underestimates the requirement of support for weak rock masses particularly in Himalaya. It provides same length of rock bolts irrespective of rock mass quality. Therefore, it would be appropriate to decide the length of rock bolts through numerical modeling or semi empirical methods *see* IS 15026.

4 Instrumentation shall be carried out in the tunnel and large underground openings to monitor the roof and wall deformations, support pressure and zone of influence around the underground opening for evaluation of supports.



Rock mass quality $Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$

Support categories

① Unsupported or spot bolting
② Spot bolting, SB
③ Systematic bolting, fibre reinforced sprayed concrete, 5-6cm. B+Sfr
④ Fibre reinforced sprayed concrete and bolting, 6-9cm sfr (E500) + B
⑤ Fibre reinforced sprayed concrete and bolting, 9-12cm sfr (E700) + B
⑥ Fibre reinforced sprayed concrete and bolting, 12-15cm + reinforced ribs of sprayed concrete and bolting, sfr (E700) + RRS I + B
⑦ Fibre reinforced sprayed concrete > 15cm + reinforced ribs of sprayed concrete and bolting, sfr (E1000) + RRS II + B
⑧ Cast concrete lining, CCA or sfr (E1000) + RSS III + B
⑨ Special evaluation

Bolts spacing is mainly based on Ø20mm
E = Energy absorption in fibre reinforced sprayed concrete
ESR = Excavation Support ratio
Areas with dashed lines have no empirical data

RRS - Spacing related to Q value

⟺ I	Si30/6 Ø16 - Ø20 (span 10m) D40/6 + 2 Ø16-20 (span 20m)
⟺ II	Si35/6 Ø16-20 (span 5m) D45/6 + 2 Ø16-20 (span 10m) D55/6 + 4 Ø20 (span 20m)
⟺ III	D40/6 + 4 Ø16-20 (span 5m) D55/6 + Ø20 (span 10m) Special evaluation (span 20m)

Si30/6 = Single layer of 6 rebars, 30cm thickness of sprayed concrete

D = Double layer of rebars
Ø16 = Rebar diameter is 16mm
c/c = RSS spacing centre - centre

FIG. 3 CHART FOR THE DESIGN OF SUPPORT FOR UNDERGROUND OPENINGS USING *Q* MEAN

5 ESTIMATION OF TUNNEL DEFORMATION OR CLOSURE

The tunnel roof and wall deformations are related with *Q* as follows (*see* equations 18 and 19):

$$\Delta_v = \frac{\text{Span}}{100\,Q}\sqrt{\frac{\sigma_v}{q_c}} \qquad \dots(18)$$

$$\Delta_h = \frac{\text{Height}}{100\,Q}\sqrt{\frac{\sigma_h}{q_c}} \qquad \dots(19)$$

where

Δv and Δ_h = roof and wall deformations respectively [units would be similar to tunnel span or height];
= radial tunnel roof and wall deformation or radial closure(u_a);
σ_v and σ_h = *in-situ* vertical and horizontal stresses respectively, in MPa; and
q_c = UCS of intact rock material, in MPa.

NOTE — The tunnel deformation or closure estimated by equations 18 and 19 are for tunnels without supports.

6 CRITICAL STRAIN

The critical strain (ε_{cr}) is defined as an empirical value of tangential strain (ε_θ) at the periphery of the opening above which the squeezing problems are likely to be encountered. Equation 20 is suggested for estimating the critical strain of jointed rock mass.

$$\varepsilon_{cr} = 5.84 \frac{q_c^{0.88}}{Q^{0.12} E_r^{0.63}} \text{ (percent)} \geq \frac{100\, q_c}{E_r} \quad \dots (20)$$

$$\varepsilon_{cr} > \varepsilon_\theta = 100\, u_a/a$$

where

ε_{cr} = critical strain in percent;

Q = (post-construction) rock mass quality;

q_c = unconfined compressive strength of intact rock material, in MPa;

E_r = modulus of deformation of intact rock material, in MPa;

U_a = radial tunnel closure in roof; and

a = equivalent radius of tunnel or cavern.

The ratio of observed strain to the critical strain may then be used to quantify the squeezing potential or squeezing index (*SI*).

$$SI = \frac{\text{Observed or expected strain } (u_a\,/\,a)}{\text{Critical strain } (\varepsilon_{cr})} \quad \dots (21)$$

The strain actually occurring at the periphery of the opening may be obtained through numerical modelling or through observations and analysis of the field data.

Using the SI value the squeezing potential (*SI*) is obtained as given in Table 11.

Table 11 Classification for Squeezing Potential in Tunnels

(*Clause* 6)

Sl No.	Class Number	Squeezing Level	SI
(1)	(2)	(3)	(4)
i)	1	No Squeezing (NS)	SI < 1.0
ii)	2	Light Squeezing (LS)	1.0 < SI ≤ 2.0
ii)	3	Fair Squeezing (FS)	2.0 < SI ≤ 3.0
iv)	4	Heavy Squeezing (HS)	3.0 < SI ≤ 5.0
v)	5	Very heavy squeezing (VHS)	5.0 < SI

7 ENGINEERING PROPERTIES OF ROCK MASS

Some of the engineering properties of the rock mass can also be estimated using the *Q*-value as follows.

Modulus of deformation varies considerably. It is more in the horizontal direction than in the vertical direction. However, a mean value of static modulus of deformation (*Ed*) in GPa, can be obtained by using the following relation in the underground openings:

$$E_d = 10 \left(\frac{Q.q_c}{100} \right)^{1/3} < E_r \text{ [for } Q = 0.1 \text{ to } 100 \text{ and } q_c = 10\text{-}200 \text{ MPa]} \quad \dots (22)$$

where

E_r = modulus of deformation of intact rock in GPa.

The uniaxial compressive strength of rock mass is correlated as follows (for *SRF*=2.5 in equation 23):

$$q_{cmass} = 7\gamma\, (Q)^{1/3} \quad \dots (23)$$

(for $Q < 10$, $100 > q_c > 2$ MPa, $SRF = 2.5$ and $J_w = 1.0$)

where

γ = rock density or unit weight of the rock mass, in t/m^3 or g/cc; and

q_{cmass} = rock mass compressive strength, in MPa.

The uniaxial compressive strength of rock mass is found to be approximately independent of *UCS* of rock material for $Q < 10$ (for $SRF = 2.5$).

For the purpose of calculation of q_{cmass} using equation 23, *SRF* shall be assumed as 2.5, irrespective of actual ground condition. This is because rock mass in failing condition will have $SRF = 2.5$. Further, it should be realized that there is significant strength enhancement of rock mass in tunnels and underground openings due to shorter length of rock joints and effect of intermediate principal stress along the tunnel axis. After the failure of the rock mass, for example in squeezing ground conditions, the *SRF* will be as high as 20.

Following correlations (equations 24 and 25) are suggested to obtain the peak cohesive strength (c_p) and peak angle of internal friction or frictional strength (ϕ_p) of the rock mass in the underground openings:

$$c_p = \left(\frac{RQD}{J_n} \right) \left(\frac{1}{SRF} \right) \left(\frac{\sigma_c}{100} \right) \text{ MPa} \quad \dots (24)$$

$$\phi_p = \tan^{-1} \left(\frac{J_r}{J_a} \text{x} J_w + \mathbf{0.1} \right) \text{ degree} \quad \dots (25)$$

The cohesive strength (c_p) represents the component of the rock mass requiring shotcrete or mesh or concrete support. Similarly angle of internal friction or frictional strength (ϕ_p) represents the component of the rock mass requiring the bolting. The rock masses with low c_p values require more shotcrete, whereas rock masses with low ϕ_p values require more rock bolts. The cohesion from equation 24 is drastically much higher than cohesion from RMR which is applicable to rock slopes, bridge and dam foundations. This anomaly is due to shorter lengths of rock joints inside underground openings.

The permeability (k) of the rock mass is roughly given by equation 26 at 20°C.

$$k \approx \left(\frac{0.002}{Q}\right)\left(\frac{100}{JCS}\right)\left(\frac{1}{\mathrm{H}^{5/3}}\right) \text{ m/s} \qquad \ldots(26)$$

where

Q = *in-situ* rock mass quality ($Q = 0.1$ to 100);
= $(RQD/J_n)(J_a/J_r)(J_w/SRF)$ [*see* equation 1];
JCS = joint wall compressive strength, in MPa, and
H = depth of a point under consideration below ground surface.

NOTES

1 The judgment of engineer-in-charge/designer shall prevail in case of estimation of various engineering properties of rock masses.

2 The rock mass with permeability more than 10^{-5}cm/s is groutable.

ANNEX A

(*Foreword*)

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